MATERIAL CHARACTERIZATION AND OTHER THICKNESS DESIGN CONSIDERATIONS FOR AIRFIELD PAVEMENT RUBBLIZATION

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ABSTRACT

This paper is based on preliminary work done under Airfield Asphalt Pavement Technology Program Project (AAPTP) 04-01, "Development of Guidelines for Rubblization" [1]. While the project covers all design, construction and quality control aspects for rubblization of airfield concrete pavements, this paper focuses on thickness design considerations. Only the layered elastic design methodology is addressed in this paper, even though the project also addresses the CBR method for military airfield design. A large portion of this paper summarizes backcalculated moduli values of rubblized concrete layers from projects found in the literature (performed by other researchers). New backcalculations of FWD data from other rubblized projects were performed by the authors to obtain additional moduli. These values are summarized, analyzed and compared to recommended ranges of rubblized material published elsewhere. A relationship between rubblized modulus and slab thickness is examined. The "retained modulus" concept is explored utilizing the same data. Minimum overlay thickness criteria are discussed by considering practical issues such as compaction, smoothness and profile. Finally, conclusions are provided.

INTRODUCTION TO RUBBLIZATION

Rubblization is the process of fracturing the existing Portland Cement Concrete Pavement (PCCP) in-place into small-interconnected pieces that typically serve as a base course for a new Hot Mix Asphalt (HMA) overlay. By fracturing the existing PCCP into small pieces, the underlying slab integrity and movement is eliminated that would cause reflective cracking. Any existing pavement layers below the concrete slabs, such as a crushed aggregate or stabilized base, remain in place to provide additional structural support for a new HMA pavement. Since these layers remain and there are no hauling or disposal costs, rubblization is a cost-effective rehabilitation method.

Rubblizing PCCP should result in the complete destruction of any slab action before applying the HMA overlay. For concrete pavement with steel, such as jointed reinforced concrete pavements (JRCP) and continuously reinforced concrete pavement (CRCP), the concrete-to-steel bond should generally be broken. This is necessary to eliminate slab action that could cause reflective cracking. Rubblization reduces the existing PCCP into crushed aggregate base with a high degree of particle interlock. In the cases of thicker slabs, the interlock can be so great in the lower half of the slab that only micro-fissures separate the PCC pieces.

There are two basic types of rubblization equipment; the resonant pavement breaker (RPB) and the multi-head breaker (MHB). Both types can vary in size and weight depending on the model. The RPB is shown in Figure 1 and the MHB is shown in Figure 2. Fitts described the rubblization process with the RPB [2] while Thompson did the same with the MHB [3] at TRB in 2005. The RPB is a self-propelled breaking unit that produces low amplitude (1/2-1 inch), high frequency (42-46) impacts per second (hertz) through a massive steel beam, often described as a "giant tuning fork." The vibrating foot rubblizes the concrete pavement in narrow strips as the machine moves forward along the unfractured edge of the pavement. The MHB is both a tractor and breaking unit, consisting of numerous pairs of 1,200 lb, 8-inch wide hammers mounted laterally across the breaking unit that produces continuous breakage from side to side. Each hammer pair operates independently and develops between 1,000 and 8,000 foot-pounds of energy (depending upon the drop height selected) and cycles at a rate of 30 to 35 impacts per minute. The tractor travels on the unbroken slab as it moves forward and tows the breaking unit behind. Production rates for both the RPB and the MHB are generally from 6,000 to 8,000 yd^2 per unit per workday shift, with thick airfield pavements being on the low end of this range.

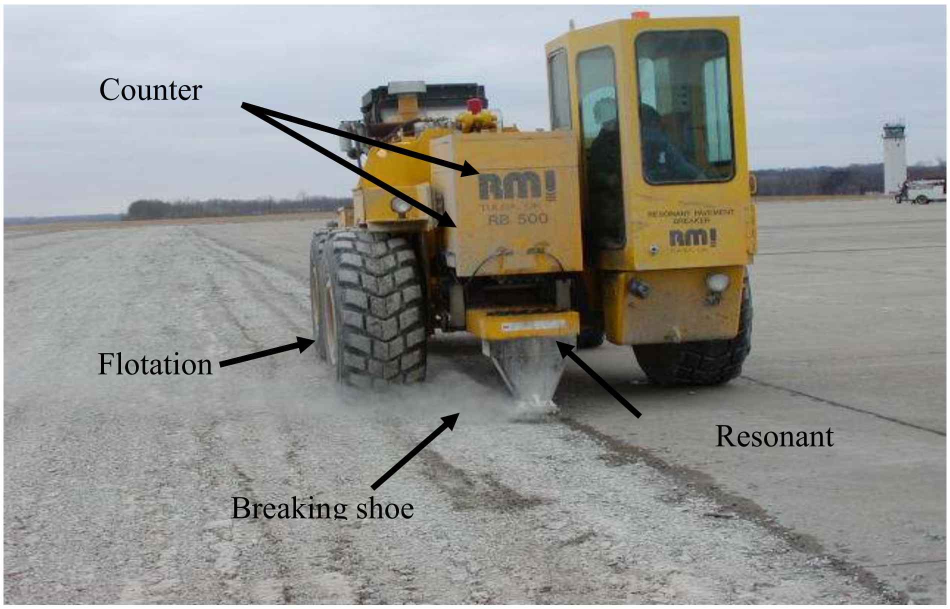

Figure 1. Resonant Pavement Breaker (RPB)

Figure 2. Multiple Head Breaker (MHB)

Rubblization technology and equipment were initially developed for highway pavements, but there is growing interest in using this technology for rehabilitating airfield PCCP. Approximately one-half million square meters of airfields had been rubblized prior to 2005, compared to more than 41 million square meters of highways. Buncher and Jones summarized the state of the art with airfield rubbilization in 2005 [4].

LAYERED ELASTIC DESIGN WITH RUBBLIZATION

Since rehabilitation using rubblization essentially converts a deteriorating rigid pavement system into a new flexible pavement, a new flexible pavement structural design should be performed to determine the total new HMA thickness (placed in a series of lifts). Some designers have used a crushed aggregate base or recycled concrete layer over the rubblized PCC prior to the HMA being placed. Whether this is part of the design or not, the rubblized layer must be characterized. When using a layered elastic design method, an elastic modulus (E) is chosen. Guidance is needed to help the designer select an appropriate modulus value for the rubblized layer, and criteria must be established for determining a minimum HMA overlay thickness.

The Federal Aviation Administration's (FAA) mechanistic-empirical layered elastic design method for flexible pavements is described in Chapter 7 of FAA Advisory Circular 150/5320-6D. Computerized versions of this procedure are embodied in the FAA's LEDFAA Program. The U.S. military's layered elastic procedure is called LEEP and is encased in the PCASE suite of pavement design software. LEEP is relatively similar to LEDFAA where pavement layers are defined by the elastic modulus, Poisson's ratio, thickness, and layer interface (all layers assumed to be fully bonded for flexible pavements). Rubblized layers have typically been characterized as an unbound crushed stone base, (P-209 material) which typically has a modulus of 50 to 60 ksi.

FAA Engineering Brief 66 [5] provides guidance for rubblizing PCC pavements. When using the CBR design method, the guidance states, "most rubblized material will perform equal to or better than P-209 material. Unless additional project specific information is available, a one-to-one substitution should be used…" For the layered elastic method, EB-66 references the guidance in the Asphalt Institute MS-17 manual [6] stating that a design modulus for the rubblized layer should be selected from field data at a 95% reliability factor. Since designs are typically performed without any rubblized field data, EB-66 goes on to say, "rubblized pavement moduli have been found to vary from a low of 30 ksi to over 300 ksi, depending on slab thickness, base type and condition of base layers. Pavements that are less than 9 inches and have marginal bases and subgrade conditions will be at the low end of the modulus range. Thicker pavements over 9 inches with good bases or stabilized bases have shown much higher modulus values. Pavements constructed by pre WW II methods typically have low modulus values." EB-66 goes on to say larger size fractions and any bonded steel reinforcement will produce higher modulus, but if too high then the possibility of reflective cracking may exist.

AVERAGE MODULUS VALUES FROM OTHERS

This section provides a summary of the literature review for backcalculated modulus values of rubblized layers on past projects. Backcalculated moduli of the subgrade were also examined when available in the literature to determine if a significant change occurs between pre and post rubblization.

Witczak and Rada Study: Witczak and Rada reported at TRB in 1992 [7] on the first nationwide performance comparison of the various methods for fracturing PCC slabs; rubblization, crack and seat (C&S) and break and seat (B&S). For the 22 rubblization sections included in the study, the backcalculated moduli ranged from 200 to 700 ksi, with a mean value of 412 ksi and standard deviation of 154 ksi (CV of 37%). Comparatively between the three processes, both the average moduli and the variability were highest for the B&S sections, followed by the C&S sections, and lowest for the rubblized sections. This is intuitiveley logical because particle size is much smaller with the rubblization relative to the C&S and B&S, and the reinforcing steel inherent with B&S sections should increase stiffness.

Selfridge ANG Runway: Anderson reported in 2004 [8] on the project to rubblize the Selfridge ANG Base runway in 2002. HWD deflection data were collected to evaluate the moduli of the layers before and after rubblization. Some results are shown in Table 1.

Table 1.
Pre and Post Moduli for Concrete at Selfridge ANGB [8]

Station	Slab Thickness inches	PCC Modulus x 10^6 (psi) Avg./ Std.Dev.	Rubblized PCC Modulus, x 10^5 (psi) Avg./ Std.Dev.
117+50-127+00	16	4.37/ 2.62	4.30 / 1.58
127+00-160+00	21	2.73 / 2.28	3.70 / 1.96
160+00 – 170+00	13	2.05 / 0.57	1.83 / 3.77

Anderson concluded that “it is evident that the rubblized layer is an order of magnitude less than the concrete, namely 180,000 to 430,000 psi as compared to 3 million psi for the concrete. For comparison, a typical high quality crushed aggregate base has a modulus of 30,000 psi, and hot mix asphalt of about 400,000 psi.”

US Army Corps of Engineers Report: The Corps of Engineers actively monitored three rubblization projects: Selfridge ANGB runway, Hunter Army Airfield taxiway and Niagara Falls Air Reserve Station runway. The results were published in 2005 [9]. For the Selfridge project, the reported backcalculated moduli were substantially higher than those reported by Anderson (same project) as discussed above. For the Hunter project, Antigo Construction reported it was a Crack and Seat project and not rubblization. The relatively high modulus value reported by the Army for a 6-inch slab supports Antigo’s report that this was C&S. For the Niagara Falls project, the average backcalculated moduli for a 9.5-inch slab reportedly ranged from 101 to 156 ksi.

Illinois DOT I-57 Project: The Illinois DOT rubblized a 10-inch PCC section of I-57 in 1990. Details can be found in a 1999 TRB paper by Thompson [10]. FWD data was collected on an annual basis for 7 years. From data presented in this paper, Figure 3 was developed showing the increase in backcalculated moduli of the rubblized layer. The cause of the increase in rubblized modulus with time is not known. From Thompson's work, the average modulus calculated for all sections was 134 ksi.

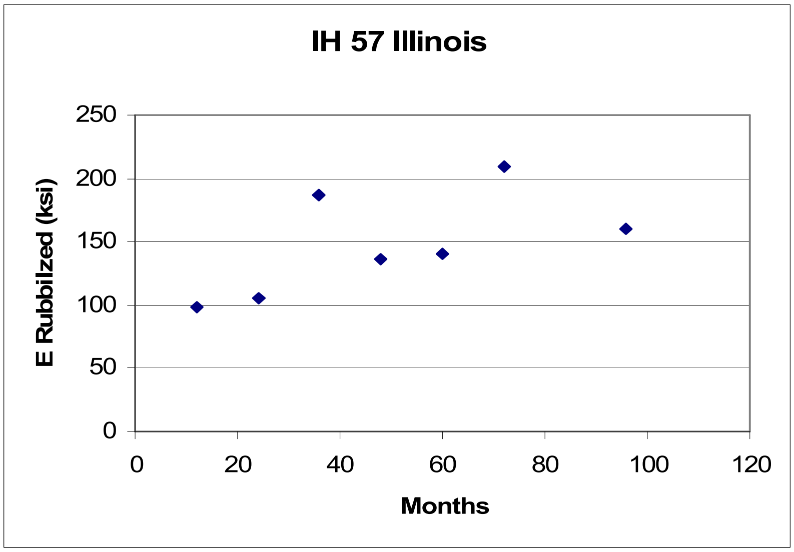

Figure 3. Change in Rubblized Modulus with Time, I-57 Illinois [10].

Michigan DOT Report: The most complete set of deflection data available was assembled and processed by Baladi and Svasdisant in a study published in 2002 [11] on the performance of rubblized pavements in Michigan. In their Phase II study, they conducted field and laboratory testing as well as backcalculation and mechanistic modeling. The pavements in Michigan are largely 9 inch JRCP with temperature steel at mid-depth. They concluded that the concrete in these sections was not rubblized uniformly with depth. Above the temperature steel, the pavement was broken into material with a composition similar to granular base. They referred to this top layer as "rubblized" material, but henceforth in this paper, we will refer to it as "crushed" material. Below the steel, the material was classified as fractured concrete, containing only hairline fractures in the slabs. Based on this, they explored two methods for modeling a rubblized pavement. In the first, the average modulus of the entire rubblized layer was determined. In the second, the rubblized layer was divided into two layers and an estimate was made of the stiffness of each layer. This was not done through normal backcalculation because of the difficulty in modeling two relatively thin layers (4.5 inches each) and getting reasonable results. Instead, an analysis procedure was developed using the deflection bowls and a set of regression equations. The authors of this paper found that modeling the rubblized material as two separate layers with distinctly different moduli significantly lowered the calculated fatigue life.

A separate long-term performance study was later conducted by Applied Research Technologies on the initial MI projects using the MHB included in the Baladi 2002 study. This study was reported on at the 2007 TRB [12] and found no correlation between the reported breakage pattern by Baladi and the ultimate long-term pavement performance, which was overall very good. The study concluded that the eventual long-term performance was much more related to the presence of under-drains than the extent of initial breakage. HMA overlay thicknesses on these projects were between 5.5 to 10.5 inches, with the average about 7.5 to 8.0 inches.

Detroit Metropolitan Airport Trial: Two short test sections at the Detroit Metropolitan Airport were rubblized and overlaid in 2006 to determine the applicability of using either the RMI or MHB equipment to rehabilitate the runway. Both the RMI section and the MHB section consisted of 17-inch thick PCC with wire reinforcement (located 4 inches below surface) on top of 8 to 9 inches of HMA base. HWD testing was completed in May of 2006 and the results were summarized in a letter report by Kohn [13]. For the RMI section, the average backcalculated modulus was 229 ksi with a standard deviation of 110 ksi. For the MHB section, the average was 113 ksi with a S.D. of 42 ksi. These moduli are somewhat less then those reported from other projects (i.e. Selfridge AFB) with similar slab thickness. Our own backcalculation analysis verified similar low modulus values for these sections. The test pits dug in the MHB section verified that the slab had broken full depth, but test pits dug in the RMI section showed only partial and uneven breakage, accounting for the higher variability with the RMI on this project.

Indiana DOT US 41: Galal reported at TRB in 1999 [14] on Indiana DOT efforts to obtain layer moduli on two sections of US 41 which were rubblized in 1991. The first section was 8 inches of CRCP and the second was 10 inches of JRCP. The pre-rubblized moduli were 3,882 and 3,692 ksi, respectively. After rubblization, the reported average moduli were 181 ksi for the CRCP section (standard deviation of 54 ksi) and 166 ksi for the JRCP section (SD of 28 ksi).

FAA's National Airport Pavement Test Facility: A complete discussion of the testing, analysis and findings on the test sections that were rubblized in 2005 at the FAA National Airport Pavement Test Facility (NAPTF) will be included in the Final Report of AAPTP Project 04-01. This write-up will be limited to summarizing the backcalculation of the rubblized layers at NAPTF. Garg explained the three distinct test sections designated MRC, MRG and MRS [15]. All three sections consisted of 12 inches of PCCP with no reinforcing steel except dowels at the joints. All were built over a medium strength subgrade with a CBR range of 7-8. The rubblization occurred with a RPB, then all sections were overlaid with 5 inches of HMA (P-401 material). MR**C** had a 10" **conventional** crushed stone subbase (P-154 material), MR**G** had slabs directly on **grade** (no base or subbase), and MR**S** had a 6" **stabilized** base layer of econocrete (P-306 material) over a 6" subbase layer (P-154 material).

Average pre-fractured PCC moduli for each section were determined with FWD testing. After rubblization and placing the HMA, but prior to trafficking, FWD testing was again performed. The average pre-traffic rubblized moduli for the three sections were:

- MRC: 371 ksi
- MRG: 586 ksi
- MRS: 289 ksi

The coefficient of variability (CV) of the rubblized moduli at individual stations was about 30%. The subgrade moduli in these sections did not change significantly from before rubblization.

These average rubblized moduli are somewhat high compared to other projects of similar slab thicknesses. This may be explained by examining the typical rubblized material excavated from one of the post-trafficked trenches shown in Figure 4. The top 2-3" of the rubblized PCC was typically broken down to particle sizes of 1-inch minus, while the particle sizes in the bottom 9" of the rubblized layer typically had dimensions as large as 12 to 30 inches. The slabs were fractured full depth but not to the gradation limits typically required in most rubblization specifications. The fractures were hairline and there was a high degree of particle interlock below the top 3 inches of "crushed" material. NAPTF personnel estimated that approximately 80% of the dowels examined in the trenches remained bonded to the concrete.

Figure 4. Excavated Rubblized PCC in MRG Section at NAPTF (15)

Rubblization Project in Chile: Rubblization technology (using the RMI) has been available in Chile since 2004. A study of a 5 km pilot project found the backcalculated modulus of rubblized material to be 2.7 to 3 times greater than of aggregate base with a CBR >80% [16].

AVERAGE MODULUS VALUES FROM NEW BACKCALCULATIONS

In conducting AAPTP Project 04-01, several FWD data sets were assembled by the authors to evaluate post-rubblized backcalculated moduli. The following data were processed using the MODULUS 6 software [17].

Texas DOT US 83: US 83 Highway in Childress District consists of a thickened edge 9-6-9 pavement which was rubblized in 2004 with the RPB. A 0.6-mile monitoring site was established and FWD data was collected before rubblization and then 6 and 18 months after placement of the HMA. In performing the backcalculations, a uniform slab thickness of 8 inches was assumed. The following are the average backcalculated modulus results.

- Intact PCC before Rubblization: 3,072 ksi
- 6 months after Rubblizing: 114 ksi
- 18 months after Rubblizing: 199 ksi

The rubblized section is performing very well. The sandy subgrade did not show a significant change in modulus after rubblization.

Michigan DOT I-75: The MDOT report by Baladi in 2002 [11] that has already been discussed provides extensive FWD data on numerous rubblized sections in Michigan. One conclusion was that the slab action was not always completely destroyed nor was the temperature steel always completely debonded. As a consequence, MDOT has tightened enforcement of their specification that requires all steel to be 100% debonded. According to some rubblization contractors, this means additional passes of their equipment are required and the results are that the top layer can become “powdered.” This additional effort may have detrimental impact on the structural capacity of the pavement, as is supported by the analysis below.

Backcalculations were performed by the authors of FWD data taken from the Baladi report [11] on two I-75 sections rubblized prior to 2000. Backcalculations were also performed by the authors of FWD data provided by MDOT on two comparable I-75 sections rubblized in 2005, which occurred under MDOT’s tighter enforcement of their rubblization specification. One of the 2005 sections was rubblized with the MHB and the other with the RPB. The PCC thickness was 9 inches on all four sections. Rubblized modulus information from each of these sections is shown in Table 2.

Table 2.
Moduli of Rubblized Layers on I-75.

Year Rubblized	I–75 Site Location	Average Rubblized Modulus (ksi)	Standard Dev. (ksi)	Number of Deflections
Pre-2000	Test Site 1	225	70	40
Pre-2000	Test Site 2	188	47	40
2005	06111 MHB	57	28	75
2005	06111 RPB	53	12	55

The results in Table 2 provide an indication that substantially different moduli can be obtained depending on the level of effort and the enforcement of the rubblization specification. Repeated passes of the rubblizing equipment causes additional fracturing of the concrete, possibly losing interlock and eventually turning the slab into the equivalent of a granular base. The modulus values of 57 and 53 ksi are in the range of a granular base. This could have considerable implications on the fatigue life of the structure. It is recognized that this analysis and the subsequent conclusions are considerably limited because this was not a controlled experiment and the sections were of different ages at the time of testing. The 2005 FWD data was collected shortly after rubblizing while the pre-2000 FWD data was taken from sections rubblized several years prior. There has been some indication on other projects that the rubblized stiffness can increase with time.

LTPP Sites in Illinois: Rubblized pavements were included in the Special Pavements Studies of the LTPP program [18]. The FWD data collected on two Illinois sites (170663 and 170664) were extracted from the LTPP database and processed. Both sites had 10-inch thick slabs. The RPB was used on both.

The FWD data extracted from the LTPP database was collected before rubblization and at regular intervals afterwards for almost 8 years. The deflection data was relatively uniform, indicating even support to the HMA layer. The average rubblized moduli values (calculated from 26 to 30 deflection bowls per site) are plotted versus time in Figure 5. It should be noted that all the rubblized modulus values are at least twice as large as modulus values typically used for crushed stone base, and that there appears to be an increase of rubblized modulus over time.

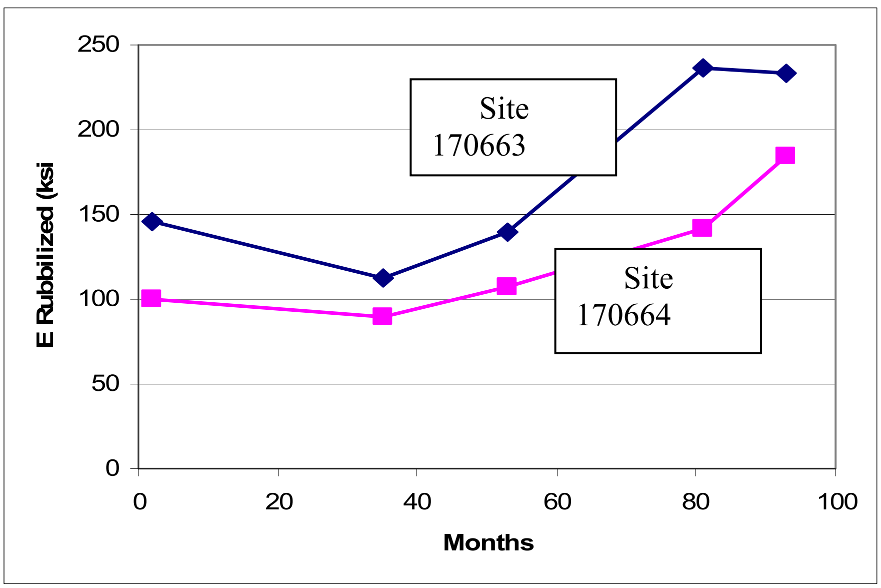

Figure 5. Plot of Rubblized Layer Moduli versus Time from Illinois LTPP Sites

SELECTING A DESIGN MODULUS FOR THE RUBBLIZED LAYER

It is clear from the data presented that a wide range of rubblized modulus values exist, given various factors such as slab thickness, particle size, rubblization energy applied, reinforcing steel, etc. At one end is the most recent data from Michigan indicating that repeated passes of the MHB or RMI can break the existing slabs down to a granular base with moduli in the 50-60 ksi range. At the opposite end are the results from the NAPTF where large PCC pieces were interlocked and dowel bars not always debonded, resulted in moduli in the 300-600 ksi range.

Slab thickness appears to be the logical variable to examine to help predict an in-place modulus of a rubblized layer for project design. Relationships between moduli and variables unknown at the time of design, such as average particle size from a test pit, would not be helpful. Slab thickness appears to indirectly relate to two factors that should influence in-place modulus; particle size and degree of interlock. For thicker slabs, rubblized particles tend to be larger and interlocked stronger, leading to a higher modulus. For thinner slabs often built on poor support, the unstable conditions can cause poor particle interlock leading to a lower modulus.

A summary of the average initial backcalculated modulus values for rubblized layers in the pavement sections already discussed in this paper are tabulated in Table 3, along with the corresponding slab thickness. For this data set, a very low value and a very high value were excluded because the authors felt they were not representative of a "typical" rubblized section.

Table 3.
Summary of Initial Backcalculated Moduli of Rubblized Layers.

Author Reference (Site)	Slab Thickness (inches)	Backcalculated Moduli (ksi)
Anderson (Selfridge)	16	430
Anderson (Selfridge)	21	370
Anderson (Selfridge)	13	183
Velez-Vega (Niagara Falls)	9.5	156
Velez-Vega (Niagara Falls)	9.5	101
Thompson (IDOT)	10	134
Galal (INDOT)	8	181
Galal (INDOT)	10	161
Kohn (Detroit Airport)	17	229
Kohn (Detroit Airport)	17	113
McQueen (NAPTF)	12	371
McQueen (NAPTF)	12	289
*Authors (US 83-Texas)	8	114
*Authors (I75-Michigan)	9	225
*Authors (I75-Michigan)	9	188
*Authors (LTPP-Illinois)	10	146
*Authors (LTPP-Illinois)	10	100

*Authors of this paper

The extreme low value was from the recent (2005) I-75 data in Michigan, where additional passes were necessary for both the RPB and MHB to completely debond the steel, resulting in the top of these rubblized layers being "powderized" according to a contractor. The extreme high value was the MRG section at NAPTF, which had 24-inch size pieces revealed from the test pit. Placing PCC slabs directly on grade (MRG) is not representative of airfields with slabs 12 inches thick. It is possible that the lack of base or subbase on MRG had an energy dampening effect with the RPB, resulting in larger pieces.

The 17 data points in Table 3 represent moduli representative of typical airport pavements. They are plotted in Figure 6 to show the relationship between modulus and slab thickness.

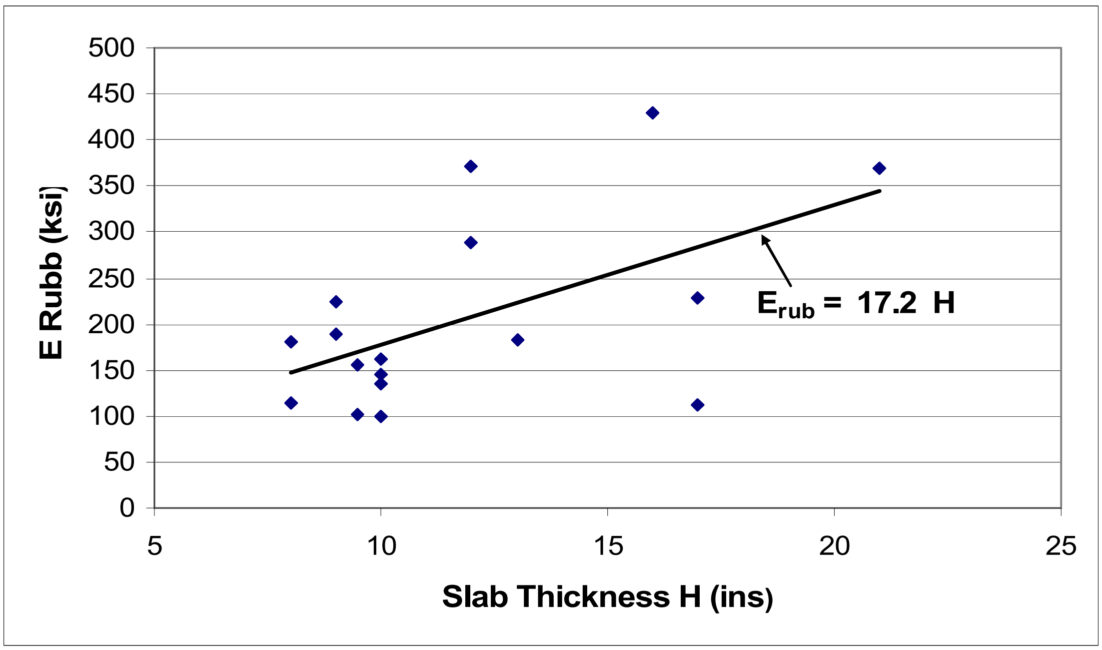

Figure 6. Average Initial Moduli versus Slab Thickness for All Sections Discussed.

Using regression techniques, the following equation was developed:

$E_{Rubb} = 17.2\ H \qquad R^2 = 0.32$

where

E_{Rubb}: Backcalculated modulus of the rubblized concrete (ksi)

H: PCC slab thickness (inches)

It is recognized that this correlation ($R^2 = 0.32$) is poor, and would be worse if one or both of the extreme values were included. For example, including the MRG value only would lower the R^2 value to 0.22. On the other hand, removing the lowest value from Detroit AP (113 ksi) would raise the R^2 value to 0.5.

Besides this collection of backcalculated values, there are several publications which have provided recommended modulus ranges for rubblized PCC. These are summarized below.

- Witczak & Rada Paper published in 1992: 200 to 700 ksi with mean of 412 ksi
- Asphalt Institute's MS-17 Manual published in 1999: at least 250 ksi typical
- FAA's EB-66 published in 2004 [5]: 30 to 300ksi (low end of this range for thin slabs and high end of this range for thick slabs)
- PerRoad Users Guide published in 2006 [19]: 300 to 700 ksi with 500 ksi typical
- New AASHTO M-E Design Guide for Highways in 2004 [20]: 150 ksi (highways typically 8 to 12 inches thick.

Comparing these published recommended ranges with the collection of backcalculated values summarized in Table 3, it appears the FAA's recommendations and those of the new AASHTO M-E Design Guide may be the most appropriate. The others appear slightly high relative to the findings here.

In conclusion, it is evident from the data presented in this chapter that the design practice of characterizing rubblized PCC as a crushed stone base (50-60 ksi) is rather conservative. Since the range of moduli for rubblized PCC has been found here to be 100-400 ksi, it is more accurate to characterize rubblized PCC as a new or different material category with a modulus in the range of a flexible asphalt base (which has a range in LEDFAA of 150-400 ksi). This seems logical since a rubblized slab is a combination of "crushed" and "fractured" concrete. The following ranges are suggested for design on airfelds:

- For slabs 6 to 8 inches thick: Moduli from 100 to 135 ksi
- For slabs 8 to 14 inches thick: Moduli from 135 to 235 ksi
- For slabs >14 inches thick: Moduli from 235 to 400 ksi

The data in Table 3 are <u>initial</u> modulus values. There were four studies cited whose data indicates some level of increase in the backcalculated moduli for a rubblized layer over time. Unsubstantiated theories for this increase include further compaction or some type of re-cementing that may occur among the broken PCC. Since current design procedures do not accommodate for changing moduli over time, and due to the limited nature of the data indicating this trend, the authors recommend this not be a consideration when selecting a design modulus.

"RETAINED MODULUS" CONCEPT

The "retained modulus" concept can help predict a rubblized modulus (for design purposes) as a percent of the unbroken PCC modulus. Initial estimates by Witczak [7] of "retained modulus" for highway pavements were in the 8% to 10% range. Of the projects already discussed in this paper whose pre-fractured PCC moduli were known, a data set is compiled in Table 4. Retained modulus values range from 1.8% to 13.5%, with an average of 6.0%.

Table 4.
Summary of "Retained Modulus" Values

Author Reference (Site)	Slab Thickness (inches)	Pre - Rubb. Modulus (ksi)	Post Rubb. Modulus (ksi)	Retained Modulus (%)
Anderson (Selfridge)	16	4370	430	9.8
Anderson (Selfridge)	21	2730	370	13.5
Anderson (Selfridge)	13	2050	183	8.9
Galal (INDOT)	8	3882	181	4.7
Galal (INDOT)	10	3692	166	4.5
NAPTF (MRC)	12	3895	371	9.5
NAPTF (MRS)	12	4871	289	5.9
*Authors (US 83-TX)	8	3072	114 (i)	3.7
*Authors (US 83-TX)	8	3072	199 (f)	6.5
*Authors (LTPP-IL)	10	5975	146 (i)	2.4
*Authors (LTPP-IL)	10	5975	233 (f)	3.9
*Authors (LTPP-IL)	10	5618	100 (i)	1.8
*Authors (LTPP-IL)	10	5618	184 (f)	3.2

*Authors of this paper

This data is plotted as "retained modulus" versus slab thickness in Figure 7. The relationship shows that thicker slabs provide a higher percentage of "retained modulus." The correlation (R^2 = 0.69) is fair, but the data is recognized to be very limited for drawing conclusions. Of the 13 data points, three of them represent an initial value (i) where a paired final value (f) determined later on the same section is also included. Thus, Figure 7 represents only 10 different sections. In comparison, Figure 6 represents 17 different sections. The concept of "retained modulus" seems promising and should be explored further as more data becomes available. When considering the presence of steel in the PCC slabs, the concept intuitively makes sense since steel should cause an increase in the moduli of PCC both before and after rubblization.

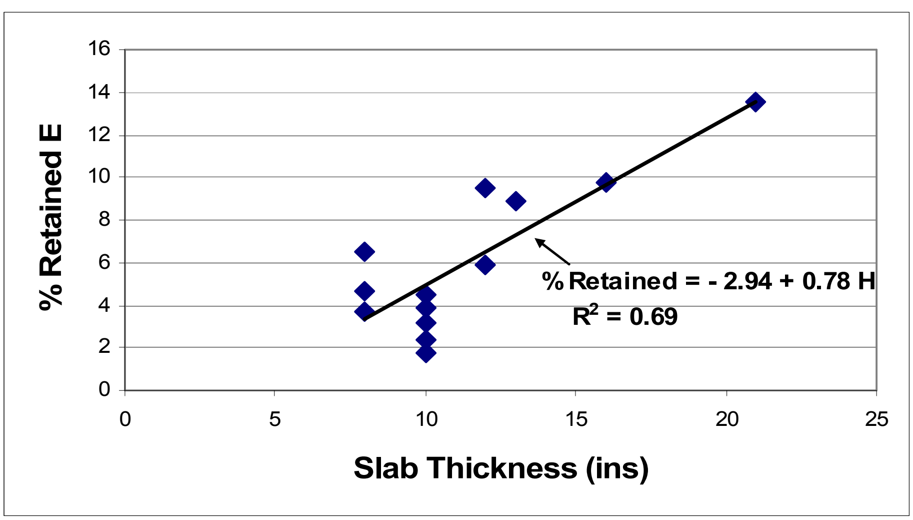

Figure 7. "Retained Modulus" Percent versus Slab Thickness.

MINUMUM OVERLAY CRITERIA

While a mechanistic analysis might suggest that the minimum HMA overlay thickness required to prevent subgrade distortion is minimal in some cases, construction considerations come into play when considering a minimum thickness criterion. The ability to successfully place and compact the overlay to meet specified compaction and smoothness requirements must be considered. The Asphalt Institute MS-17 states: “most agencies have accepted 5 inches as a minimum overlay thickness” over rubblized PCCP. FAA (EB-66) currently requires “a minimum 4-inch asphalt surface course” for pavements designed for aircraft with gross weights of less than 30,000 pounds and a minimum of 5 inches for all other cases.

Correspondence and conversation with paving contractors who have worked on highway projects using either rubblization type suggest that at least two lifts of HMA are necessary to meet grading and smoothness requirements. The contractors also state that the first lift must be at least 3 inches due to factors inhibiting the ability to achieve the desired level of compaction. The surface of rubblized material is loose relative to a P-209 base material because of the lack of fine particles, which hinders the HMA compaction process by not allowing sufficient confinement of the HMA. Since applying a prime coat to the rubblized surface is not recommended because of the lack of surface fines, sufficient lift thickness is necessary to prevent slippage between the HMA and the rubblized surface during compaction. A minimum of 3 inches is also necessary to level the profile and minimize the possibility of FOD from construction traffic running on the first lift. A minimum of two lifts is necessary because the rubblized and rolled surface is usually quite rough, and at least two lifts are necessary to meet smoothness requirements. With this the case, the minimum recommended overlay thickness should be 5 inches, since the thinnest surface lift recommended is 2 inches using mixture gradations presently available in the FAA’s P-401 or P-401 Superpave specifications. It is also relevant to note that no project examined in this research had a final overlay thickness less than 5 inches.

There have been airfield projects where an unbound base, such as crushed aggregate or crushed concrete, has been placed directly over rubblized and rolled PCCP prior to placing the HMA. As with an HMA base course, the ability to place and compact an unbound base to meet in-place criteria should govern the minimum layer thickness. The FAA P-208 (Aggregate Base Course) specification requires a lift thickness between 3 and 6 inches, while P-209 (Crushed Aggregate Base Course) sets a maximum compacted lift thickness of 6 inches. When any unbound base is placed directly on top rubblized/rolled material, the authors here recommend a minimum unbound layer thickness of 4 inches. Mixing of this unbound layer directly on the rubblized/rolled surface is discouraged because problems could occur with disturbing the larger rubblized pieces at the surface with the grading equipment. Placing this unbound layer with an HMA paving machine is recommended to achieve greater smoothness and avoid unnecessary disturbance. Placing a prime coat on this unbound base is recommended prior to placing HMA even though priming is not recommended directly on rubblized material. The difference is the lack of fines and variable surface texture that typically exist on a rubblized surface. When an unbound base is placed over a rubblized layer, the minimum HMA thickness requirement over that unbound material should apply. For general aviation airfields, this criterion is currently two 1½-inch lifts for a total of 3 inches for airfields receiving aircraft under 30,000 lbs.

Care must be taken when selecting lift thicknesses and mix designations (by particle size) to ensure sufficient lift thickness for adequate particle reorientation during compaction. Otherwise, sufficient density will not be achieved and/or the aggregate in the mix will be crushed during the rolling operation. In either case, the minimum overlay thickness requirement should be the absolute minimum used on the project. This is an important design consideration where significant leveling and variable overlay thicknesses are required.

CONCLUSIONS

Based on the data, analysis and discussion presented, the following conclusions are made:

- Moduli of in-service rubblized material was found to be in the 100-400 ksi range, significantly higher than that of crushed aggregate base material with a typical range of 50-60 ksi.

- Rubblized modulus (E) appears to be influenced by slab thickness (t), although there was a low R^2 correlation factor between E and t. Thicker slabs tended to have higher modulus.

- Rubblized modulus appears to be related to the pre-rubblized PCC modulus. Retained modulus values (E_{rub} / E_{pcc}) were found to generally be in the range of 3 to 10%. The thicker slabs tended to have higher "retained modulus" values versus the thinner slabs. The presence of reinforcing steel in the PCC should increase both the pre and post-rubblized modulus.

- There was an indication on some projects that the rubblized modulus appeared to increase with time.

- Rubblized modulus is dependent on the level of rubblization. Repeat runs of either rubblization equipment should reduce the final moduli.

- Heavily reinforced slabs can cause concern regarding effectiveness of rubblization, as there can be minimal breakage below the steel. That being said, there were no rubblized projects found in the literature that have any reflective cracking in the asphalt overlay caused from unfractured PCC slabs thought to be rubblized.

- Regarding subgrade moduli, there was no noted change in subgrade strength before and after rubblization.

- Regarding the two types of rubblization equipment, there were no obvious trends or differences in rubblized moduli between the MHB and the RPB. Each type at different times and various sections produced higher, lower, and similar moduli relative to the other.

- If placing HMA directly over rubblized material, the <u>minimum</u> HMA overlay thickness is 5 inches, placed in two lifts with the first lift being at least 3 inches. If an unbound layer is placed directly over the rubblized surface, the minimum unbound layer thickness is 4 inches, and the minimum HMA overlay thickness criteria existing for that unbound layer should apply.

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REFERENCES

1. Request for Proposal for Airfield Asphalt Pavement Technology Program Project 04-01, "Development of Guidelines for Rubblization", http://www.aaptp.us/Project04-01RFP.html
2. Fitts, G., "Rubblization Using Resonant Frequency Equipment", TRC Circular E-C087, TRB, Jan 2006.
3. Thompson, M., "Rubblization Using Multi-head Breaker Equipment", TRC Circular E-C087, TRB, Jan 2006.
4. Buncher, M. and Jones, W., "Rubblization of Airfield Pavements - State of the Practice," TRC Circular E-C087, TRB, Jan 2006.
5. Engineering Brief No. 66, *Rubblized Portland Cement Concrete Base Course*, Federal Aviation Administration, February 2004.
6. Manual Series No. 17, Asphalt Overlays for Highway and Street Rehabilitation, Asphalt Institute, 1999.
7. Witczak, M. and Rada, G. "Nationwide Evaluation Study of Asphalt Concrete Overlays Placed on Fractured Portland Cement Concrete Pavements", Transportation Research Record 1374, pp 19 to 26, 1992.
8. Anderson, J., Thompson, M., Schilling D., Kohn, D. and Shinners, G., "Runway Rubblization at Selfridge ANG", paper at DoD's Transportation Systems Workshop, Fort Lauderdale FL, 2004.
9. Velez-Vega, E., "Rehabilitation of Airfield Concrete Pavement using the Rubblization Procedure", paper presented at the Tri-Service Infrastructure System Conference, Aug, 2005.
10. Thompson, M., "Hot Mix Asphalt Design Concepts for Rubblized Portland Cement Concrete Pavements", Transportation Research Record 1684, 1999.
11. Baladi, G. and Svasdisant, T., "Identify Causes for Under Performing Rubblized Concrete Pavement Projects – Phase II, Volumes I and II (Appendices)", Michigan State University, MDOT Research Report RC-1416, August 2002.
12. Wolters, A., Smith, K., and Peterson, C., "Evaluation of Rubbilized Pavement Sections in Michigan", pre-print paper for Transportation Research Board, January 2007.
13. Kohn, S., "Nondestructive Testing Rubblized Test Sections, Taxiway W1 DTW, SME Project PP52499" Letter Report from Soil and Materials Engineers Inc to Northwest Airlines, July 2006
14. Galal, K., Coree, B., Haddock, J. and White, T., "Structural Adequacy of Rubblized PCC Pavements," Transportation Research Record 1684, 1999.
15. Garg, N., "Post – Traffic Results", PowerPoint presentation at FAA Airport Pavement Working Group Meeting, Atlantic City, NJ, February 2006.

16. Thenoux, G., González, M., and Halles, F., "Rehabilitación y Reciclado de Pavimentos de Hormigón Mediante la Técnica de Pulverización/Trituración (Rubblizing) y Recapado Asfáltico," paper presented at the Trigésimo Cuarta Reunión del Asfalto, November 2006, Mar del Plata, Argentina; Comisión Permanente del Asfalto, Argentina.
17. Liu, W. and Scullion, T. "Users Manual for MODULUS 6.0 for Windows," TTI Report 0-1869-2, November 2001.
18. Ambroz, J. and Darter M., "Rehabilitation of Jointed Concrete Pavements: SPS 6, Initial Evaluation and Analysis," FHWA-RD-01-169, Oct 2005.
19. "*A Guide to PerRoad*," Users Manual for PerRoad-Version 3.0, Pavement Thickness Design Analysis Software for Designing Perpetual Pavements, www.asphaltalliance.com , 2006.
20. ARA, Inc, "Guide for Mechanistic Empirical Design of New and Rehabilitated pavements", March 2004.